
DRAINAGE REPORT

FOR

WHITE MTN. ESTATES

Assessor's Parcel No.: 26-240-10

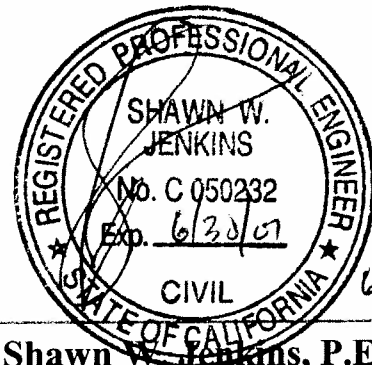
Owner: Bob Stark

MONO COUNTY, CALIFORNIA

Revised
October 2005
June 2006

PREPARED BY

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CIVIL ENGINEERING & CONSTRUCTION SERVICES

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I. GENERAL LOCATION AND DEVELOPMENT DESCRIPTION

Location of Property:

The proposed development is located off of California State Route 6 on White Mountain Estates Road and is shown in the attached **Figure 1**. The assessor's parcel number is 26-090-38. The site sits along the western base of the White Mountains, adjacent to the existing White Mountain Estates Subdivision. Proposed access to the site will be through an extension of the existing Tungsten Road to the south and Tenaya Drive to the north.

Description of Property:

The area of development is approximately 76.6 acres in size at an average elevation of about 4400' above sea level. The neighboring infrastructure consists of an approximately 20 acre single-family rural residential subdivision, which lies downstream from the proposed project. The groundcover consists of native sagebrush and tumbleweeds with a sandy loam soil type with gravel. The topography is shown on the Site Plan (*Sheet 1*) in the Drainage Plans, and exists primarily of an east to west slope at approximately 7% to 10% grade. There are numerous existing drainage courses throughout the project site that stem from steep mountain canyons to the east.

Project Description:

The project intent is to construct a subdivision consisting of approximately 45 new lots ranging in size from 0.5 acres to 8.10 acres, including common areas equaling an approximate 30.86 acres combined. The lots will serve as single-family rural residential homes and shall be constructed in two phases. Phase one shall consist of the construction of lots 1-38, 45, and phase two will include the construction of lots 39-44 as shown on the tentative tract map. Major site work shall include the grading and construction of new paved roadways consisting of 6" compacted subgrade, 6" compacted aggregate base, and 3" of asphalt concrete. Other site work activities include the installation and construction of necessary drainage structures including ditches, swales, and culverts, as well as housing structures within the individual lots. Existing access roads will be utilized and/or realigned where applicable to maintain access to existing facilities and mountain trail accessibility.

II. DRAINAGE BASIN DESCRIPTION

Drainage Description:

The project site lies along the western base of the White Mountains. Numerous canyons drain towards the valley floor that can pose substantial erosion and flooding concerns during strong storm events.

The project site lies within the path of two major canyons, which are off-chutes of the Piute Canyon and Coldwater Canyon Drainage Basins. Historically, it is believed that

Coldwater Canyon was the main source for runoff entering the project area, and that these drainage patterns have been altered over the years by geological events as well as severe storm occurrences. The current contributing canyons are split into 3 sub-basins, as shown on the Drainage Basin Map (*Figure 2*), and analyzed separately to estimate the design flow of runoff. The contributing sub-basins are approximately 1820 acres in size combined, with groundcover consisting of hard rock outcrops, and steep, dense sandy washes which can create an extensive amount of runoff. The sub-basin flows (*A, B&C*) enter the project site as a combination of overland and ditch flow at several non-defined natural washes along the northeastern property boundary. The exact flow patterns are unpredictable in this area and can be deceiving to the eye, however it is assumed that the majority of the flow from Basins A enters the more “defined” gullies and ditches located on the eastern portion of the property. From there they appear to flow in a southwest direction through Lot 48 and common area Lot D having little if any effect on the proposed project. These flows are assumed to maintain their natural drainage course and discharge off the property near the southeast corner of Lot 38. Flows from Basin A will not affect areas of new construction and shall maintain their natural paths, in turn, requiring no storm drain system. The majority of the flows from Basin B appear to enter the project near the northern terminus of Tuolumne Road. It is assumed that only a portion of the total flow from this drainage basin will enter the project site as depicted by site topography and physical washes encountered during the investigation. These flows will have an impact on the project and will require adequate drainage control structures to protect a majority of the lots and roadways along the northern border of the property. Basin C, an off-chute of Basin B, is primarily overland and sheet flow. The flows from Basin C, combined with onsite flows within the northeastern corner of the project site, are assumed to enter the project site through the canyon near the terminus of Wadkins Place.

The project site along with the contributing drainage sub-basins are classified as having Type II Rainfall and Soil Type B, which are used in the SCS Graphical Peak Discharge method of computing storm water runoff. Many assumptions were made to carry out the necessary calculations to estimate storm water runoff based on familiarity of the area and engineering judgment. These can be seen in the peak runoff calculations shown later in this report.

The 25-year/24-hr and 100-year/24-hr flows for the contributing basins are summarized below in Table 1.

Table I			
Basin	Flow Rates (ft ³ /s)		
	Q ₂₅	Q ₁₀₀	Design Flow Rate
A	366	814 *	0
B	20	51 **	45
C	7	19 ***	30
Total	393	884	

*Flow does not impact project; not used in design flow calcs

**Approximately ¼ of this flow impacts the project

***Full amount of this flow assumed to impact project

The onsite runoff was estimated using the Rational Method for both pre-development and post-development conditions for the 25-year flood event as previously required by Mono

County. These flows were used to determine the amount of additional runoff generated by development. The total amount of additional runoff was found to be about 14 ft³/s. However, a previous review by Mono County indicates that detention for the additional 14 ft³/s would not be required and therefore calculations for detention design are eliminated from this study.

The onsite flows were split into separate sub-basins and analyzed to determine post-development runoff flows contributing to specific areas throughout the project as shown on the Onsite Drainage Schematic (*Figure 3*) attached to this report. Drainage control structures within these areas were sized based upon the individual flows contributing to the facility.

Previous Drainage Studies:

The California Department of Transportation has conducted similar drainage studies for this area in the past. Findings from a Caltrans hydrology report dated January 20, 1975 suggest there is potential for extreme washouts and/or flooding in areas along the base of the White Mountains and Highway 6 due to the immense canyons discharging into the Chalfant, Hammil and Benton Valleys.

III. PROPOSED DRAINAGE FACILITIES

General Description:

The intent of the proposed drainage system is to minimize and control storm runoff entering the site. The majority of the storm runoff flows through the property to the southeast and discharges without causing any specific threat to the proposed structures. Other flows entering the project flow directly across many of the proposed lots and roadways. To control this flow and protect future residents from floods and erosion, proposed drainage ditches capable of handling the design storm shall intercept the offsite flows at the upstream end of Wadkins Place (*Ditch A*) as well as along the northern perimeter of the property (*Ditch B*), and be directed through the project where they will discharge into the existing drainage system at Ponderosa Street. Other control structures will include PCC 'Arizona' Swales designed to accommodate the design storm flows, roadside grader ditches designed to accommodate the on-site runoff at specific locations, and necessary corrugated metal culverts, to accommodate passage of ditch runoff across roadways. The proposed drainage easements will utilize the existing flow paths and topography as much as possible to reduce the amount of excavation and optimize flow transport. Drainage details are shown in the calculations section of this report.

Hydrologic Criteria:

Precipitation occurs as both snowfall and rainfall. History shows that most of the major flooding problems have been caused by short duration, high intensity rainfall events, such as thunderstorms and cloudbursts. There are reports (*Caltrans Highway 6 Hydrology Report 1975*) of storms causing walls of mud and water up to 6 feet high flowing from

several of the adjacent canyons near this project site. Therefore it was decided that the drainage design be based upon rainfall rather than snowmelt.

The precipitation data used to compute design rainfall for this project came from both Point Precipitation Frequency and Intensity-Duration-Frequency Estimates from the National Weather Service NOAA Atlas 14. A nearby observation site located at California 37.408°N, 118.294° near Silver Canyon (*elev: 4977 located south of the project near the base of the White Mountains*) was used and assumed to have similar rainfall characteristics as the basins contributing to this project. Runoff rates for the 25-year/24-hr and 100-year/24-hr storms were estimated using data from the Silver Canyon observing site based on an annual maxima series.

Using the data from the Silver Canyon site we were able to estimate precipitation depths and frequencies for both the 25-year and 100-year storms events. The precipitation depths for the 24-hour storm, which is required for SCS Method, were 3.55 inches and 4.94 inches for the 25-year and 100-year storms, respectively. The corresponding intensities for the 25-year storm were used to compare the on site pre/post-development flows utilizing the Rational Method.

Runoff Calculations:

The SCS Graphical Peak Discharge method was utilized for the estimation of storm runoff occurring in the contributing drainage sub-basins since the area analyzed was larger than 80 hectares. The Rational Method was deemed more suitable to use for the estimation of the onsite flows, since the area was not as large and relatively uniform in nature. Calculations were based on the methodology set forth in the Federal Highways, "HDS 2-Highway Hydrology" Manual, which is referenced in the Caltrans Highway Design Manual. Formulas included in this manual were followed and used to estimate the runoff. Calculations were carried out in metric units and later converted to U.S. customary units.

SCS Input:

Q = depth of direct runoff (mm)
P = depth of precipitation (mm)
I_a = initial abstraction (mm)
S = maximum potential retention (mm)
CN = curve number (f(soil group, cover complex, antecedent moisture condition))
q_p = peak discharge in m³/s
q_u = unit peak discharge m³/s/km²/mm of runoff
A = drainage area in square kilometers
V = velocity (m/s)
T_c = time of concentration (hr)
C₀, C₁, C₂ = Coefficients
k = intercept coefficient used in the velocity method for time of concentration
s = slope (%)

$Q = (P - 0.2 * S) / (P + 0.8 * S)$
 $S = 25.4 * [1000 / CN - 10]$
I_a = 0.2S
 $V = k * s^2$

$$T_c = \Sigma[(L_i/V_i*60)/60 \text{ min/hr}]$$

$$q_u = 0.000431*10^{(C_0+C_1*\log(T_c)+(C_2*(\log(T_c))^2)}$$

$$q_p = q_u*A*Q$$

SCS Coefficients

Type II Rainfall			
I/P	C ₀	C ₁	C ₂
0.1	2.55323	-0.61512	-0.16403
0.3	2.46532	-0.62257	-0.11657
0.35	2.41896	-0.61594	-0.0882
0.4	2.36409	-0.59857	-0.05621
0.45	2.29238	-0.57005	-0.02281
0.5	2.20282	-0.51599	-0.01259

k	Land cover/flow regime
0.076	Forest with heavy ground litter; hay meadow (overland flow)
0.152	Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland)
0.213	Short grass pasture (overland flow)
0.274	Cultivated straight row (overland flow)
0.305	Nearly bare and untilled (overland flow); alluvial fans in western mountain regions
0.457	Grassed waterway (shallow concentrated flow)
0.491	Unpaved (shallow concentrated flow)
0.619	Paved area (shallow concentrated flow); small upland gullies

Rational Method Input:

Q = peak discharge (cfs) $Q = C*i*A$
i = rainfall intensity (in/hr)
A = drainage area (acres)
C = runoff coefficient

The rainfall intensity used in the rational method was taken from the intensity-duration-frequency graphs generated by National Weather service as mentioned above using a duration equal to the time of concentration.

Facility Design Calculations:

Despite the fact that the project site has had little flood history and receives minimal precipitation on a yearly basis, it was decided that the facility should be designed to accept incoming flows from a 100-year storm. We believe that imposing a more conservative drainage design will not only benefit the overall quality of the project but also future home-owners, property owners, designers and the surrounding community and infrastructure. Onsite runoff, however, was designed to capacitate flows from the 25-year storm since the area is relatively small in comparison to the contributing offsite drainage basins.

The 100-year design flows used for this project were taken as 45 ft³/s and 30 ft³/s for flows coming from drainage basins B and C, respectively and are shown in **Table I**. The

incoming flows from Basins B and C combined were estimated at about 70 ft³/s, in which approximately 19 ft³/s approaching the project from Basin C towards Wadkins Place and about 51 ft³/s flowing through the canyon to the northwest entering the project near the Tuolumne Road cul-de-sac. Of the 51 ft³/s coming from Basin B, it is anticipated that approximately ¾ to ½ of this flow will actually impact the project; therefore we set the 100-year design flow rate (Q_{100}) at 45 ft³/s to be conservative. All of the 19 ft³/s coming from Basin C is expected to contribute with an additional 7 ft³/s to 10 ft³/s of onsite flow from Sub-basin 1 shown in (**Figure 3**). Therefore we set the design flow for this portion of the project at 30 ft³/s. Flows from Basin A appear to travel around the limits of work and avoid any structural areas of the project.

Once the design flows were determined, adequate drainage structures were designed to accommodate these flows from the site entrance to the point of discharge off the site. Necessary calculations were made to size a drainage ditch to accommodate the off site flows entering the site for the 100-year storm. A trial and error process, which utilized Manning's equation, was used to determine appropriate ditch and swale sizes. Culvert sizes were designed using culvert design forms and nomographs. Attached is a copy of necessary calculations. Recommended sizes for ditches, swales, and culverts are attached in the *appendix section of this report*. The drainage schematic and layout can be seen on the drainage plans (*sheets D1-D6*). Ditches are designed with varied side slopes from 2:1 to 6:1 and are designed to intercept the offsite flows entering the project site. Flows shall cross the roadways through concrete swales where they will then enter the existing subdivision drainage system on Ponderosa Street and Tungsten Road where they will discharge off the property onto public land. It was found that the existing swales are adequate to accept the flows from the proposed drainage system as shown in the calculations (**Table A-3**).

Ditch A shall intercept flows in the canyon near the terminus of Wadkins Place and direct them through the project crossing both Tuolumne Road and Redwood Drive before exiting the project between Lots #2 & #45. Ditch B will intercept runoff entering the project along the northern perimeter. The ditch will start near the cul-de-sac at the northern terminus of Tuolumne Road and prevent flows from disturbing lots 24-29 by carrying them along the northern property line to the west where the ditch will make a 90 bend heading south across Redwood Drive and travel along the western property border which will protect the existing homes from onsite runoff flows from this property. The ditch will then intersect Ditch A and leave the project site through the existing drainage system in place along Ponderosa Street.

Proposed drainage easements are shown on the drainage plan as well as roadway improvement & grading plan and consist of 15', 30', 40' & 180' easements. The proposed 60-foot roadway easement serves as an element in the proposed drainage system in that the shoulders will adequately carry on-site runoff and direct it into adjacent structures (*swales, ditches, culverts, etc.*). ESE is not responsible for the design of individual driveway entrances to any of the proposed lots. It is recommended that driveway access be constructed in the form of either culverts or swales. The roadway cross-section shown on the drainage plan is designed to allow for vehicle passage across

shoulder ditch onto any lot. Finish grade contours are shown and plotted on the drainage plan which show the locations and flow paths of proposed roadside ditches.

Portions of the slopes of Ditches A and B are classified as steep slopes. The slopes combined with the design flow rates and dimensional properties of the ditch create very high velocity flows that require riprap to prevent excessive scour. To fully protect against scour and sediment transport it was found that it would take a 12"-18" mean diameter riprap, lining the entire ditch. However it was decided that due to the limited amount of material naturally available in the area, Ditch A would only be lined above Tuolumne Road and at swale inlets and outlets. Ditch B would only require riprap in the portion to the north of the Tenaya Drive swale crossing. Riprap sizing methods are based upon the method used in NCHRP Report 108 (*extension of Tractive Force Method*). It is assumed that should the design flow be reached, necessary maintenance will be required for repair to ditches and swale inlets and outlets in areas not receiving riprap.

The roadside ditches were designed to accommodate on site flows reaching 10 ft³/s. **Table A-3** shows slopes and velocities for specific on site flow rates throughout the project. It was found that the majority of the roadside ditches would require some sort of lining reinforcement to prevent excessive scour based upon recommended permissible flow velocities for unlined channels set forth in Table 862.2, of the Caltrans Highway Design Manual. It was decided to line the roadside ditches with 2"-6" cobbles which would be adequate to protect against excessive scour in locations shown on the drainage structures summary as well as on the drainage plan.

Culverts were designed and located to allow on site runoff from the proposed roadside ditches to flow through the project in the most logical fashion. It was found that one 24" corrugated metal pipe (CMP) would adequately carry the anticipated flow across Tuolumne Road at station ±0+54 and one 24" CMP crossing Redwood Drive at station ±0+31. The culvert profiles are shown on *Sheet D6* of the drainage plans. The culverts are designed with a 2% slope at lengths of approximately 63 feet and 56 feet respectively. Culvert inlets shall be graded as depressed inlets as shown by the finish grade contour lines on the drainage plans and lined with riprap. The culvert crossing Tuolumne road shall capture all flows within the east roadside ditch, preventing flow from leaving the project site along the southeast corner of the project. All flow shall leave the project site through the existing drainage system to the west.

IV. CONCLUSIONS

The proposed project lies within the path of a potentially dangerous flood zone, despite the fact that the area itself receives very little precipitation on a yearly basis. The adjacent canyons are capable of producing extremely high runoff flows, which a portion of, flow directly through the project site. These flows have the potential to wash out proposed roadways and/or flood future home sites causing severe losses. By enforcing a rather conservative drainage system, as outlined in this study, potential risks are reduced, which would protect future homes, increase public safety, secure property investments, and enhance the overall quality of the project and surrounding infrastructure.

The proposed drainage plan will have a little impact to downstream facilities. The proposed drainage structures utilize the existing topography and flow paths therefore having a minor disturbance to the natural condition. However there will be a concentration of flows, which will discharge the project into the existing drainage system at Ponderosa Street consisting of two concrete swale road crossings which in turn discharge onto public land. As advised by Mono County, the excess flow ($14 \text{ ft}^3/\text{s}$) created by developing the site will not require on-site detention. Mono County determined that an earlier detention basin design was ineffective and was eliminated from the design. (*See review by Mono County dated May 2, 2005.*) All excess runoff is designed to exit the project site through the existing drainage system on Ponderosa street located directly to the west of the project.

Drainage Ditches A & B are designed to accommodate the 100-year flood event. To control excess erosion and scour, it was found that the ditches required riprap lining consisting of a 12"-18" riprap underlined by filter fabric. However, due to the nature of the project, it was decided that the riprap lining be placed only in critical areas which may cause damage to proposed roadways, including swale inlets/outlets. It is anticipated that with or without riprap the ditches are able to handle the design discharges, but will require periodic maintenance to repair any scour that may occur. Existing roads shall be utilized as maintenance access if possible.

V. REFERENCES

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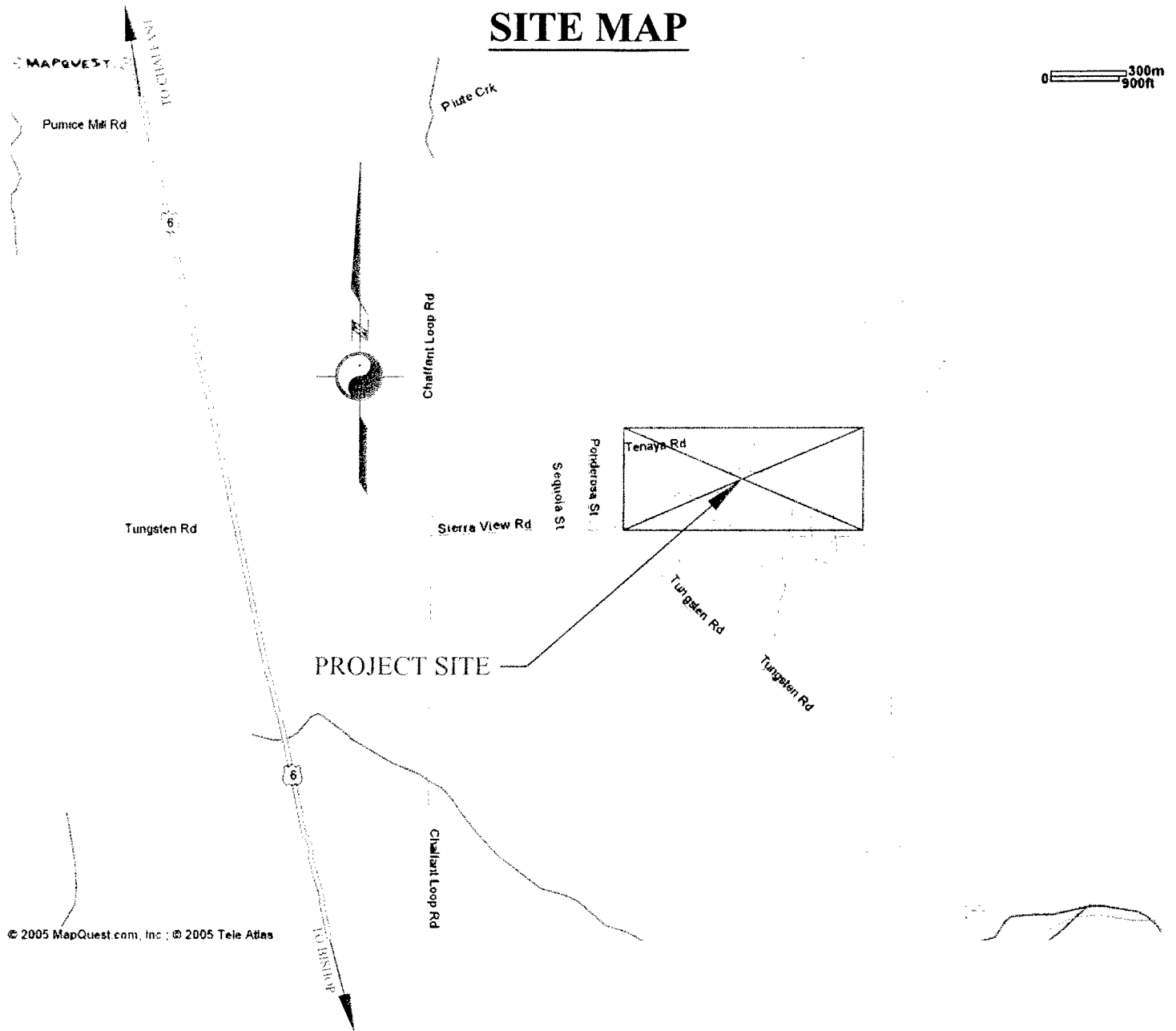
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SITE MAP



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WHITE MTN. ESTATES LOCATION MAP MONO COUNTY, CALIFORNIA

FIGURE

1 OF 3

DRAWN BY

DBB

PARCEL NUMBER

26-240-10

APPROVED

DATE

11-28-05

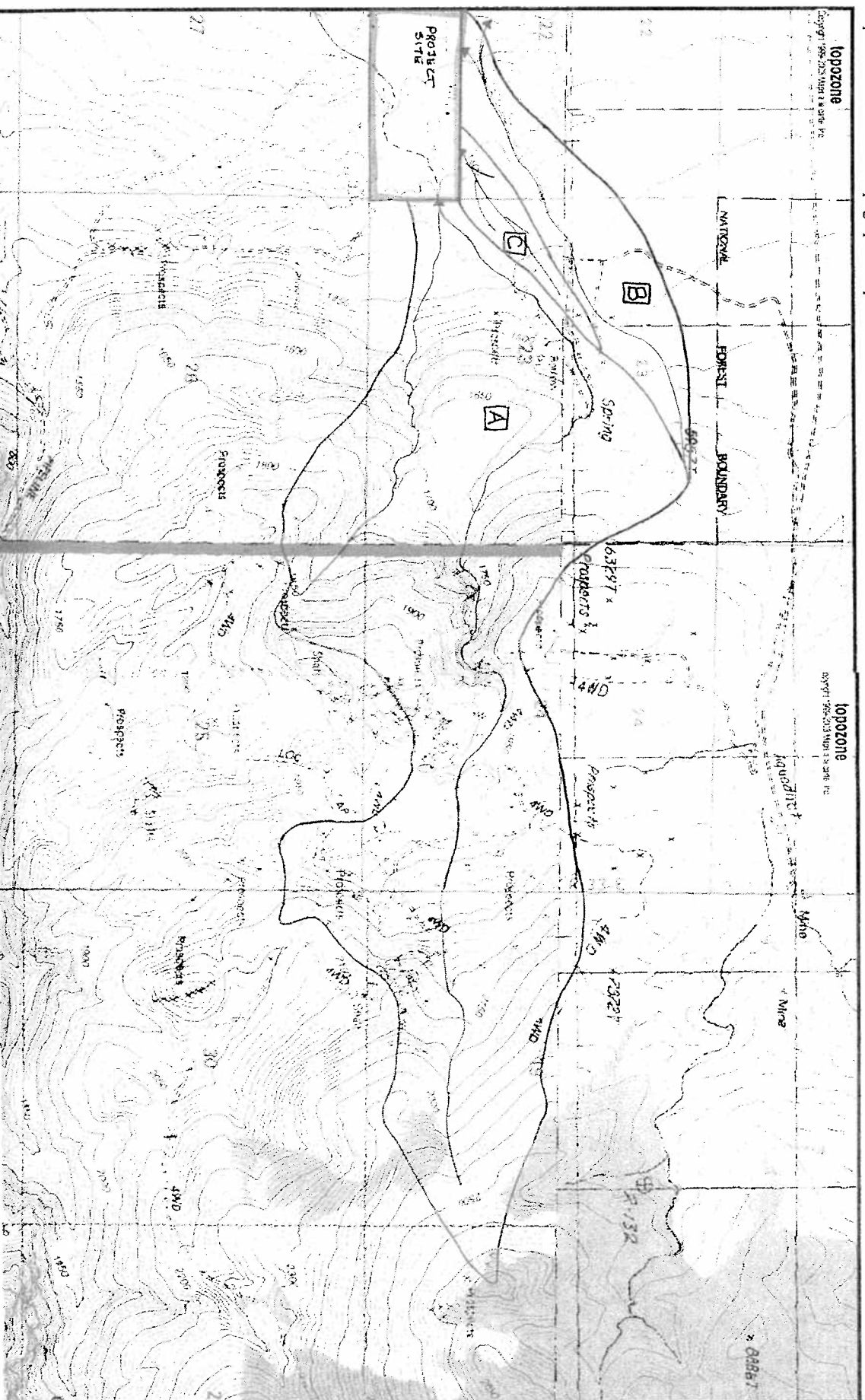
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FIGURE 2 - DRAINAGE BASIN MAP

TopoZone - The Web's Topographic Map

TopoZone - The Web's Topographic Map

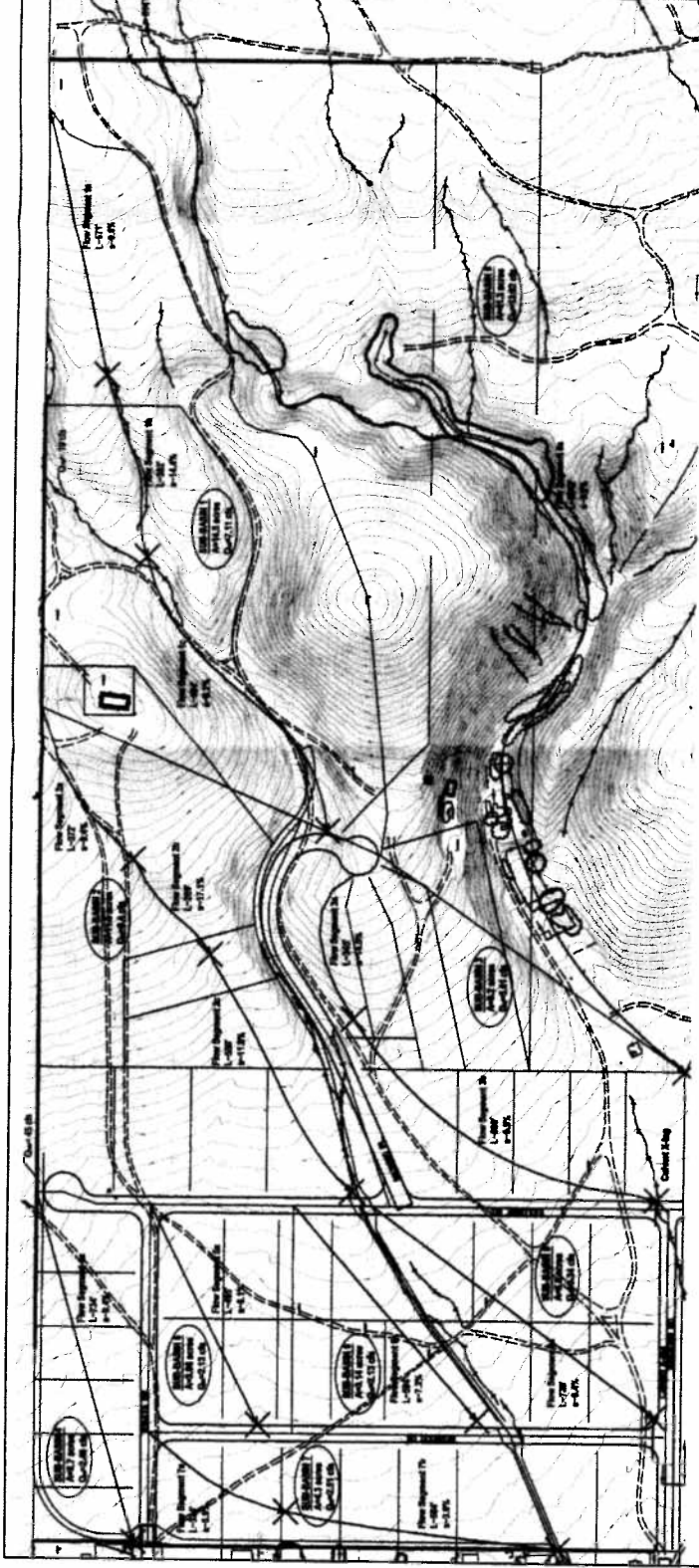


REVISIONS	DATE	BY	APP'D
1	10/1/01	W. J.
2	10/1/01	W. J.
3	10/1/01	W. J.
4	10/1/01	W. J.
5	10/1/01	W. J.
6	10/1/01	W. J.
7	10/1/01	W. J.
8	10/1/01	W. J.
9	10/1/01	W. J.
10	10/1/01	W. J.

WHITE MOUNTAIN ESTATES
ON SITE DRAINAGE SCHEMATIC
WHITE MOUNTAIN ESTATES, LLC
BOB STARK, 332 WEST HOWELL AVE.
RIDGECREST, CA 93555



PROJECT NO. 01-001
SHEET NO. F3 OF 3



White Mountain Estates Input

- A_1 = Area of Road coverage
- $A_{1,1}$ = Area of lot coverage less than 1 acre
- $A_{1,2}$ = Area of lot coverage greater than 1 acre
- A_2 = Common area coverage
- $C_1 = 0.8$
- $C_{1,1} = 0.4$
- $C_{1,2} = 0.35$
- $C_2 = 0.3$

Pre-Development Runoff:

A (acres)	C	A/C	L (ft)	s (%)	K	V (ft/hr)	T _c (hr)	I (in/hr)	Q (cfs)
70.58	0.3	22.974	3300	9	0.305	0.915	1.00	0.85	18.63

Post-Development Runoff:

Coverage	A ₁ (acres)	C ₁	A ₁ C ₁	C ₂	L (ft)	s (%)	K	V (ft/hr)	T _c (hr)	I (in/hr)	Q (cfs)
Area	5.8	0.8	5.256	0.40	3300	9	0.4	1.2	0.76	1.1	33.32
A ₂	26.37	0.4	10.548								
A ₃	27.96	0.35	9.783								
A ₄	15.65	0.3	4.695								
Σ	70.0		30.3								

Additional Runoff due to Development: 13.8 cfs



POINT PRECIPITATION FREQUENCY ESTIMATES
FROM NOAA ATLAS 14



California 37.408 N 118.294 W 4977 feet
from "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 1, Version 3
G.M. Bonnin, D. Todd, B. Lin, T. Parzybok, M. Yekta, and D. Riley
NOAA, National Weather Service, Silver Spring, Maryland, 2003
Extracted: Mon Jan 3 2005

Confidence Limits Seasonality Location Maps Other Info. Grids Maps Help Docs U.S. Map

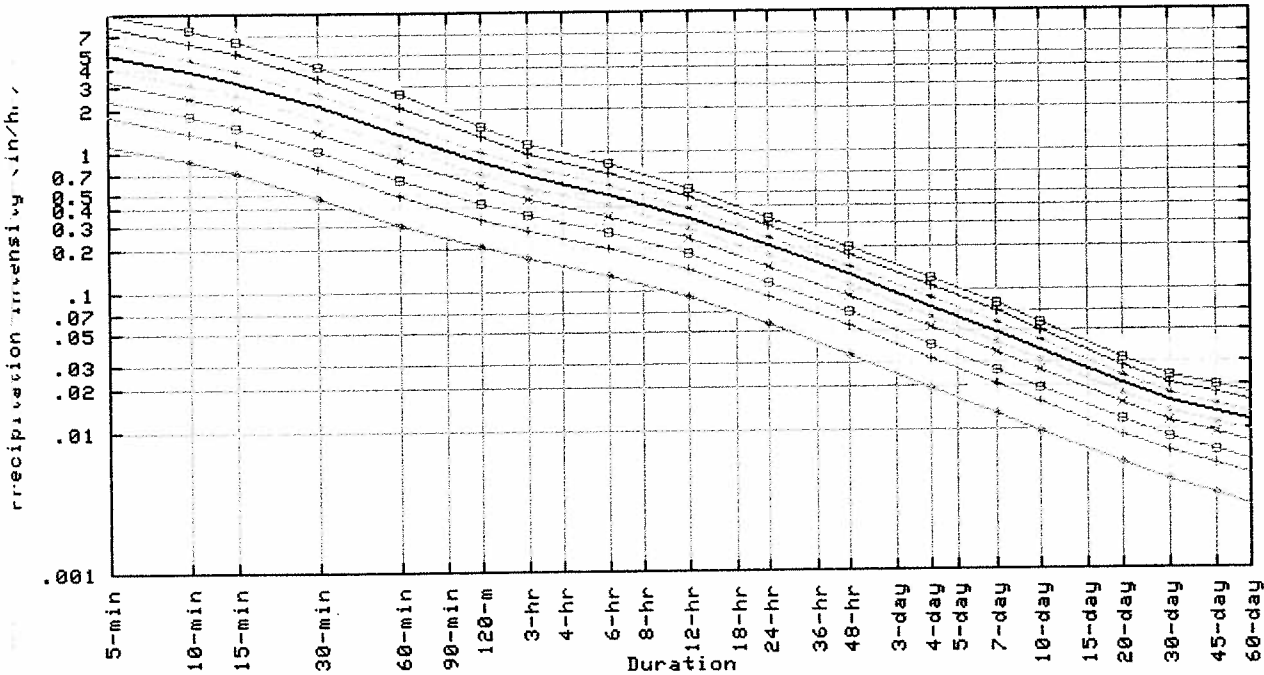
Precipitation Intensity Estimates (in/hr)

EP* (1-in-Y)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
2	1.12	0.86	0.71	0.48	0.29	0.21	0.17	0.13	0.09	0.06	0.03	0.02	0.01	0.01	0.01	0.00	0.00	0.00
5	1.82	1.39	1.15	0.77	0.48	0.33	0.27	0.20	0.14	0.09	0.05	0.03	0.02	0.02	0.01	0.01	0.01	0.00
10	2.38	1.81	1.50	1.01	0.62	0.43	0.35	0.26	0.18	0.11	0.07	0.04	0.03	0.02	0.01	0.01	0.01	0.01
25	3.26	2.48	2.05	1.38	0.85	0.57	0.46	0.34	0.24	0.15	0.09	0.05	0.03	0.03	0.01	0.01	0.01	0.01
50	4.09	3.11	2.57	1.73	1.07	0.70	0.56	0.41	0.28	0.18	0.11	0.06	0.04	0.03	0.02	0.01	0.01	0.01
100	5.04	3.84	3.17	2.14	1.32	0.84	0.66	0.48	0.33	0.21	0.13	0.08	0.05	0.04	0.02	0.02	0.01	0.01
200	6.20	4.72	3.90	2.63	1.62	1.00	0.78	0.57	0.38	0.24	0.15	0.09	0.06	0.04	0.02	0.02	0.01	0.01
500	8.07	6.14	5.08	3.42	2.11	1.26	0.97	0.70	0.46	0.29	0.18	0.11	0.07	0.05	0.03	0.02	0.02	0.02
1000	9.85	7.49	6.20	4.17	2.58	1.50	1.13	0.81	0.53	0.33	0.20	0.12	0.08	0.06	0.03	0.02	0.02	0.02

Text version of table

* These precipitation frequency estimates are based on an annual maxima series. AEP is the Annual Exceedance Probability. Please refer to the documentation for more information. NOTE: Formatting forces estimates near zero to appear as zero.

Annual Maxima based Point IDF Curves
37.408 N 118.294 W 4977 ft



Mon Jan 03 18:05:26 2005

Annual Exceedance Probability (1-in-Y)	
2-year	100-year
5-year	200-year
10-year	500-year
25-year	1000-year

Confidence Limits -

*** Upper bound of the 90% confidence interval**
Precipitation Intensity Estimates (in/hr)

AEP** (1-in-Y)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
2	1.26	0.96	0.79	0.53	0.33	0.23	0.19	0.15	0.10	0.07	0.04	0.02	0.02	0.01	0.01	0.01	0.00	0.00
5	2.01	1.53	1.26	0.85	0.53	0.36	0.30	0.23	0.16	0.11	0.06	0.04	0.02	0.02	0.01	0.01	0.01	0.01
10	2.61	1.98	1.64	1.10	0.68	0.47	0.39	0.29	0.21	0.14	0.08	0.05	0.03	0.02	0.01	0.01	0.01	0.01
25	3.58	2.72	2.25	1.51	0.94	0.63	0.51	0.38	0.27	0.18	0.11	0.06	0.04	0.03	0.02	0.01	0.01	0.01
50	4.53	3.45	2.85	1.92	1.19	0.78	0.62	0.46	0.33	0.21	0.13	0.08	0.05	0.04	0.02	0.02	0.01	0.01
100	5.62	4.27	3.53	2.38	1.47	0.94	0.75	0.56	0.39	0.25	0.15	0.09	0.06	0.04	0.02	0.02	0.02	0.01
200	6.99	5.32	4.39	2.96	1.83	1.14	0.89	0.66	0.45	0.29	0.18	0.11	0.07	0.05	0.03	0.02	0.02	0.02
500	9.33	7.11	5.87	3.95	2.45	1.46	1.13	0.82	0.55	0.35	0.22	0.13	0.09	0.06	0.03	0.03	0.02	0.02
1000	11.62	8.84	7.31	4.92	3.04	1.76	1.34	0.97	0.64	0.40	0.26	0.15	0.10	0.07	0.04	0.03	0.03	0.02

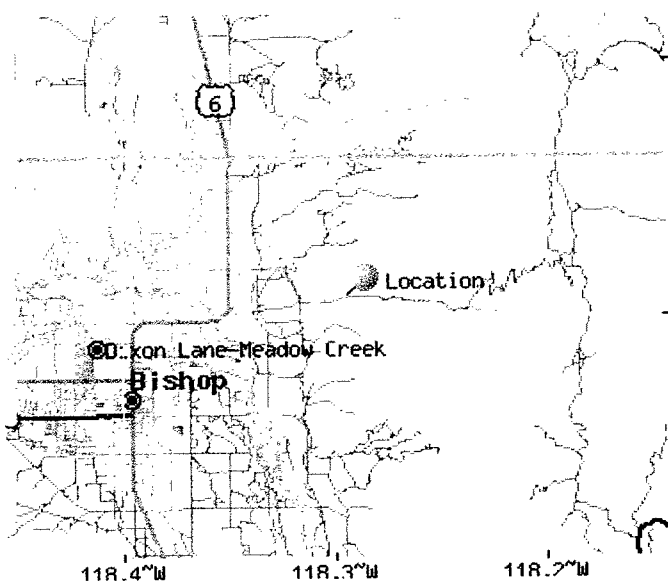
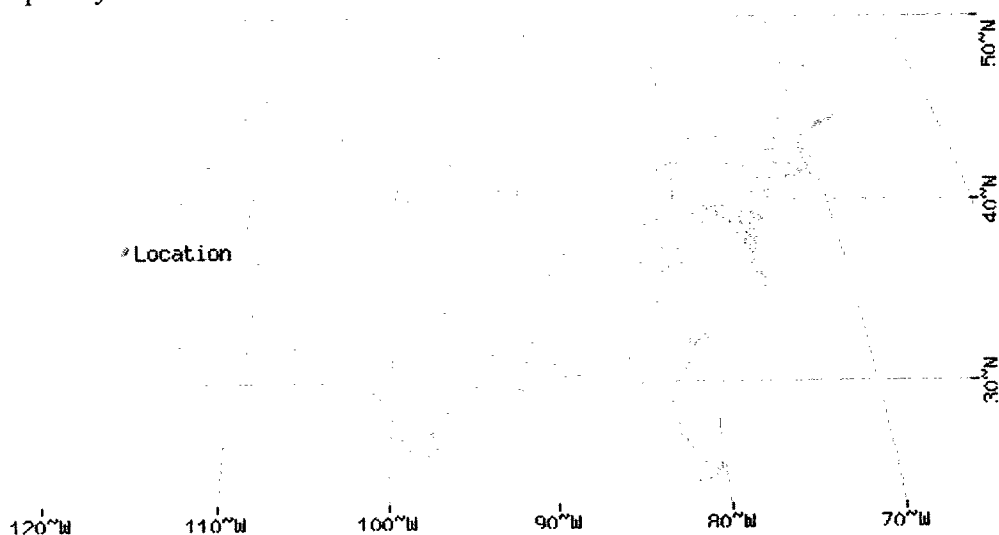
* The upper bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are greater than. These precipitation frequency estimates are based on an annual maxima series. AEP is the Annual Exceedance Probability. Please refer to the documentation for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

*** Lower bound of the 90% confidence interval**
Precipitation Intensity Estimates (in/hr)

AEP** (1-in-Y)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
2	1.02	0.78	0.65	0.43	0.27	0.19	0.16	0.11	0.08	0.05	0.03	0.02	0.01	0.01	0.00	0.00	0.00	0.00
5	1.68	1.28	1.06	0.71	0.44	0.30	0.25	0.18	0.12	0.08	0.05	0.03	0.02	0.01	0.01	0.01	0.00	0.00
10	2.16	1.63	1.35	0.91	0.56	0.38	0.31	0.23	0.16	0.10	0.06	0.03	0.02	0.02	0.01	0.01	0.01	0.00
25	2.91	2.21	1.83	1.23	0.76	0.50	0.41	0.30	0.20	0.12	0.08	0.04	0.03	0.02	0.01	0.01	0.01	0.01
50	3.57	2.72	2.25	1.51	0.94	0.60	0.48	0.35	0.24	0.15	0.09	0.05	0.03	0.03	0.01	0.01	0.01	0.01
100	4.24	3.23	2.67	1.79	1.11	0.70	0.57	0.41	0.28	0.17	0.10	0.06	0.04	0.03	0.02	0.01	0.01	0.01
200	5.02	3.82	3.16	2.13	1.32	0.82	0.65	0.47	0.32	0.19	0.12	0.07	0.04	0.03	0.02	0.01	0.01	0.01
500	6.19	4.71	3.89	2.62	1.62	0.99	0.78	0.56	0.38	0.22	0.13	0.08	0.05	0.04	0.02	0.02	0.01	0.01
1000	7.20	5.47	4.52	3.05	1.88	1.13	0.88	0.63	0.42	0.25	0.15	0.09	0.06	0.04	0.02	0.02	0.01	0.01

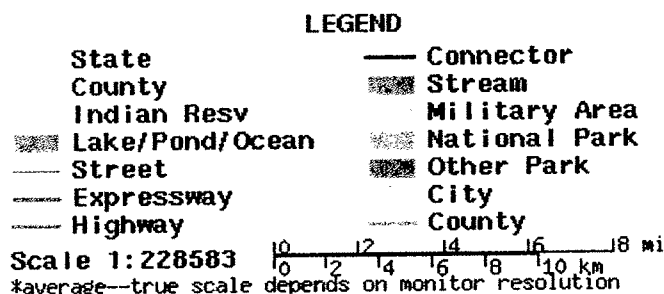
* The lower bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are less than. These precipitation frequency estimates are based on an annual maxima series. AEP is the Annual Exceedance Probability. Please refer to the documentation for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

Maps -



These maps were produced using a direct map request from the U.S. Census Bureau Mapping and Cartographic Resources Tiger Map Server.

Please read disclaimer for more information.



A new USGS digital orthophoto quadrangle (DOQ) covering this location from TerraServer; **USGS Aerial Photograph** may also be available from this site. A DOQ is a computer-generated image of an aerial photograph in which image displacement caused by terrain relief and camera tilt has been removed. It combines the image characteristics of a photograph with the geometric qualities of a map. Visit the USGS for more information.

Find the Watershed for this location using the U.S. Environmental Protection Agency's site.

precipitation frequency results are based on data from a variety of sources, but largely NCDC. The following links provide general information about observing sites in the area, regardless of if their data was used in this study. For detailed information about the stations used in this study, please refer to our documentation.

Using the National Climatic Data Center's (NCDC) station search engine, locate other climate stations within:

Table A-1: 25-year Peak Discharge Estimation Using SCS Method

Coefficients For SCS Peak Discharge Method					
Basin	Rainfall Type	Soil Type	CN	C ₀	C ₁ C ₂
A	II	B	62	2.41896	-0.61594 -0.0882
B	II	B	56	2.306722	-0.575754 -0.02949
C	II	B	56	2.306722	-0.575754 -0.02949

Drainage Basin Characteristics					
Basin	Area, A (km ²)	25 yr-24hr Rainfall, P (mm)	Max Retention, S (mm)	Initial Abstraction, I _a (mm)	I _a /P Ratio Runoff Depth, Q (mm)
A	6.8	90	156	31	0.35 16.2
B	0.46	90	200	40	0.44 10.0
C	0.12	90	200	40	0.44 10.0

Characteristics of Principle Flow Paths for Time of Concentration Estimation					
Basin	Flow Segment	Length, L (m)	Slope, s (%)	k	Velocity, V (m/s) Time of Concentration, T _c (hr)
A	1	1000	40	0.305	1.929 0.144
	2	1300	15	0.305	1.181 0.306
	3	550	15	0.305	1.181 0.129
	4	860	25	0.305	1.525 0.157
	5	1780	15	0.305	1.181 0.419
	6	580	8.5	0.305	0.889 0.181
				Total:	1.335
B	1	530	35	0.305	1.804 0.0816
	2	1650	10	0.305	0.964 0.4752
				Total:	0.557
C	1	1120	11.5	0.305	1.034 0.3008
					Total: 0.3008

Peak Discharge		
Unit Peak Discharge, q _u (m ³ /s)	Peak Discharge, q _p (m ³ /s)	q _p (ft ³ /s)
0.0943	10.360	366

0.1218	0.563	20
0.1712	0.206	7

*Total Flow entering Project site: $\frac{11.130}{393} \frac{\text{m}^3/\text{s}}{\text{ft}^3/\text{s}}$

Table A-2: 100-year Peak Discharge Estimation Using SCS Method

Coefficients For SCS Peak Discharge Method						
Basin	Rainfall Type	Soil Type	CN	C ₀	C ₁	C ₂
A	II	B	62	2.41896	-0.61594	-0.0882
B	II	B	56	2.306722	-0.575754	-0.02949
C	II	B	56	2.306722	-0.575754	-0.02949

Drainage Basin Characteristics				
Basin	Area, A (km ²)	25 yr-24hr Rainfall, P (mm)	Max Retention, S (mm)	Initial Abstraction, I _a (mm)
A	6.8	126	156	31
B	0.46	126	200	40
C	0.12	126	200	40

Characteristics of Principle Flow Paths for Time of Concentration Estimation						
Basin	Flow Segment	Length, L (m)	Slope, s (%)	k	Velocity, V (m/s)	Time of Concentration, T _c (hr)
A	1	1000	40	0.305	1.929	0.144
	2	1300	15	0.305	1.181	0.306
	3	550	15	0.305	1.181	0.129
	4	860	25	0.305	1.525	0.157
	5	1780	15	0.305	1.181	0.419
	6	580	8.5	0.305	0.889	0.181
Total:						1.335
B	1	530	35	0.305	1.804	0.0816
	2	1650	10	0.305	0.964	0.4752
Total:						0.557
C	1	1120	11.5	0.305	1.034	0.3008
	Total:					0.3008

Peak Discharge			
Unit Peak Discharge, q _u (m ³ /s)	Peak Discharge, q _p (m ³ /s)	q _p (ft ³ /s)	
0.0943	23.041	814	
0.1218	1.454	51	
0.1712	0.533	19	

*Total Flow entering Project site: $\frac{25,028}{884} \frac{\text{m}^3/\text{s}}{\text{ft}^3/\text{s}}$

Table A-3: Recommended Drainage Structures - Summary

Drainage Ditch A - Watkins Pl. - Toulumne Rd. - Design Flow of 45 cfs									
Trial Depth, y (ft)	Sideslope, m (ft/1ft)	Q (ft ³ /s)	Area, A (ft ²)	Velocity, V (ft/s)	Top Width, T (ft)	Hydraulic Radius, R (ft)	Manning's n (Riprap)	Slope, s	AR ^{2/3} nQ/1.49S ^{1/2}
1.243	4	45	6.18	7.29	9.94	0.60	0.044	9.10%	4.411
1.50	4	45	9.00	8.26	12.00	0.73	0.044	9.10%	7.281
*Indicates Normal Depth of Flow for design computation purposes									
Drainage Ditch A - Tuolumne Rd.--Redwood Dr. - Design Flow of 45 cfs									
Trial Depth, y (ft)	Sideslope, m (ft/1ft)	Q (ft ³ /s)	Area, A (ft ²)	Velocity, V (ft/s)	Top Width, T (ft)	Hydraulic Radius, R (ft)	Manning's n (Natural)	Slope, s	AR ^{2/3} nQ/1.49S ^{1/2}
0.929	4	45	3.45	13.06	7.43	0.45	0.02	8.90%	2.029
1.50	4	45	9.00	17.98	12.00	0.73	0.02	8.90%	7.281
Drainage Ditch A - Redwood Dr.--Existing Structures - Design Flow of 45 cfs									
Trial Depth, y (ft)	Sideslope, m (ft/1ft)	Q (ft ³ /s)	Area, A (ft ²)	Velocity, V (ft/s)	Top Width, T (ft)	Hydraulic Radius, R (ft)	Manning's n (Riprap)	Slope, s	AR ^{2/3} nQ/1.49S ^{1/2}
1.265	4	45	6.40	7.04	10.12	0.61	0.044	8.30%	4.622
1.50	4	45	9.00	7.89	12.00	0.73	0.044	8.30%	7.281
Drainage Ditch B - Design Flow of 45 cfs									
Trial Depth, y (ft)	Sideslope, m (ft/1ft)	Q (ft ³ /s)	Area, A (ft ²)	Velocity, V (ft/s)	Top Width, T (ft)	Hydraulic Radius, R (ft)	Manning's n (Riprap)	Slope, s	AR ^{2/3} nQ/1.49S ^{1/2}
1.265	4	45	6.40	7.04	10.12	0.61	0.044	8.30%	4.622
1.50	4	45	9.00	7.89	12.00	0.73	0.044	8.30%	7.281
Drainage Ditch B Sta 8+60- Sta End - Design Flow of 45 cfs									
Trial Depth, y (ft)	Sideslope, m (ft/1ft)	Q (ft ³ /s)	Area, A (ft ²)	Velocity, V (ft/s)	Top Width, T (ft)	Hydraulic Radius, R (ft)	Manning's n (Riprap)	Slope, s	AR ^{2/3} nQ/1.49S ^{1/2}
1.259	4	45	6.34	7.11	10.07	0.61	0.044	8.50%	4.564
1.5	4	45	9.00	7.99	12.00	0.73	0.044	8.50%	7.281

RoadsideDitch Redwood Dr. (~Sta 0+00-3+20, 3+40-10+30) : Q~10 cfs

Trial Depth, y (ft)	Sideslope, m (ft/1ft)	Q (ft ³ /s)	Area, A (ft ²)	Velocity, V (ft/s)	Top Width, T (ft)	Hydraulic Radius, R (ft)	Manning's n (Cobble Lined)	Slope, s	AR ^{2/3}	nQ/1.49S ^{1/2}
0.77	4	10	2.37	4.20	6.16	0.37	0.025	0.0185	1.23	1.23
0.75	4	10	2.25	4.13	6.00	0.36	0.025	0.0185	1.15	1.23

Roadside Ditch Tungsten Rd. (Sta 0+00-6+89): Q~10 cfs

Trial Depth, y (ft)	Sideslope, m (ft/1ft)	Q (ft ³ /s)	Area, A (ft ²)	Velocity, V (ft/s)	Top Width, T (ft)	Hydraulic Radius, R (ft)	Manning's n (Cobble Lined)	Slope, s	AR ^{2/3}	nQ/1.49S ^{1/2}
0.417	4	10	0.70	5.62	3.34	0.20	0.025	0.075	0.24	0.61
0.5	4	10	1.00	10.00	4.00	0.24	0.025	0.075	0.39	0.61

Roadside Ditch Tenaya Dr. (Sta 0+00-1+84): Q~3 cfs

Trial Depth, y (ft)	Sideslope, m (ft/1ft)	Q (ft ³ /s)	Area, A (ft ²)	Velocity, V (ft/s)	Top Width, T (ft)	Hydraulic Radius, R (ft)	Manning's n (Cobble Lined)	Slope, s	AR ^{2/3}	nQ/1.49S ^{1/2}
0.365	4	3	0.53	5.45	2.92	0.18	0.025	0.0842	0.17	0.17
0.5	4	3	1.00	3.00	4.00	0.24	0.025	0.0842	0.39	0.17

Roadside Ditch Tenaya Dr. (Sta 2+84-6+90): Q~5 cfs

Trial Depth, y (ft)	Sideslope, m (ft/1ft)	Q (ft ³ /s)	Area, A (ft ²)	Velocity, V (ft/s)	Top Width, T (ft)	Hydraulic Radius, R (ft)	Manning's n (Cobble Lined)	Slope, s	AR ^{2/3}	nQ/1.49S ^{1/2}
0.439	4	5	0.77	6.72	3.51	0.21	0.025	0.1	0.27	0.27
0.5	4	5	1.00	5.00	4.00	0.24	0.025	0.1	0.39	0.27

Existing Swale Capacity - Ponderosa St.

Approx. Depth, y (ft)	Sideslope, m (ft/1ft)	Q (ft ³ /s)	Area, A (ft ²)	Velocity, V (ft/s)	Measured Top Width, T (ft)	Hydraulic Radius, R (ft)	Manning's n (PCC Unfinished)	Est. Slope, s	AR ^{2/3}	nQ/1.49S ^{1/2}
1.5	12	350	27.00	12.94	36.00	0.75	0.015	0.025	22.24	22.28

*Flow capacity computed by setting dimensions constant and adjusting flow until normal depth occurs; (AR^{2/3} ~nQ/1.49S^{1/2}). Existing dimensions were field measured.

Manning's Equation: Q = 1.49/n * A * R^{2/3} * S^{1/2}; V=1.49/n * R^{2/3} * S^{1/2}

Q = Peak Discharge (ft³/s)
A = Area (ft²)
R = Hydraulic Radius (ft) = A/P
s = Channel Slope (ft/ft)
n = roughness coefficient
m = sideslope (ft / 1ft)
y = depth of section (ft)
A = my⁴
R = (my)/(2y*(1+m⁴)^{1/4})
T = 2my

Table A-4: Culvert Sizing to Accommodate Ditch Flow

*Design flow entering culverts from on-site drainage estimated at 10 cfs for 25-yr flood.

$$d = 1.56 [nQ/1.49s^{0.5}]^{3/8} \quad \text{*Required Diameter pipe to accommodate design flow at a fill ratio of 0.8}$$

For a corrugated metal pipe (CMP): $n \sim 0.025$

$$25\text{-yr: } d_{\text{req'd}} = 1.56[0.025 \cdot 10 / (1.49 \cdot 0.025^{0.5})]^{3/8}$$

$$d_{\text{req'd}} = 1.18 \text{ ft}$$

Use 24" CMP

Check Equation Above:

$$Q = 1.49/n \cdot A \cdot R^{2/3} \cdot S^{1/2}$$

$$A = 1/8(\theta - \sin\theta)d_o^2$$

$$\theta = 2\cos^{-1}(1-2y/d_o)$$

d_o = diameter of pipe

$$R = 1/4(1 - \sin\theta/2)d_o$$

$$y = d_o/2(1 - \cos(\theta/2))$$

$$\text{Assuming } y=0.8d_o \quad \theta = 2\cos^{-1}(1-2 \cdot 0.8d_o/d_o)$$

$$\theta = 254^\circ \quad (4.443 \text{ rad})$$

Culvert Sizing									
Q, (cfs)	θ , (degrees)	θ , (rad)	Slope, s	Manning's n (CMP)	Diameter, d _o (ft)	Commercial Size (in)	Area, A (ft ²)	R (ft)	Velocity, V (ft/s)
10	254	4.433	2.0%	0.025	1.795	24.0	1.99	0.449	5.024

$$Q = 1.49/n \cdot A \cdot R^{2/3} \cdot S^{1/2} = \quad 25\text{-yr:} \quad 10 \quad \text{ok}$$

*24" CMP would accommodate the 25-year on-site design flows.

Tractive Force Method:

$T_p = 628.3 d_{50}$ $T_d = Vys$ Setting Permissible equal to Actual: $d_{50} = Vys/628.3$

T_p = permissible shear stress (N/m^2)

d_{50} = mean diameter of lining (m)

T_d = actual channel shear stress (N/m^2)

s = channel Slope (m/m)

y = normal flow depth

γ = unit weight of water (N/m^3)

Riprap Design

Riprap Lining Design

Section	Trial Depth, y (ft)	m (ft/1ft)	Q_{25} (ft ³ /s)	A (ft ²)	V (ft/s)	T (ft)	R (ft)	n (Riprap)	Slope, s	$AR^{2/3}$	$nQ/1.49S^{1/2}$
a	1.51	2	45	4.56	9.88	6.04	0.68	0.035	9.10%	3.510	3.504
b	1.14	4	45	5.20	8.65	9.12	0.55	0.035	9.10%	3.502	3.504
c	1.155	4	45	5.34	8.44	9.24	0.56	0.035	8.50%	3.626	3.626
d	1.642	4	45	10.78	4.17	13.14	0.80	0.035	1.30%	9.267	9.271

Riprap Sizing

	Normal Depth, y (ft)	Normal Depth, y (m)	Slope, s	Allowable Shear, T_d (N/m^2)	Allowable Shear, T_d (lb/ft ²)	Rip Rap Size, d_{50} (m)	Rip Rap Size, d_{50} (in)	Rip Rap Size, d_{50} (ft)
a	1.51	0.46	9.10%	410.9	8.57	0.6539	25.7	2.1
b	1.14	0.35	9.10%	310.2	6.47	0.4937	19.4	1.6
c	1.155	0.35	8.50%	293.6	6.13	0.4672	18.4	1.5
d	1.642	0.50	1.30%	63.8	1.33	0.1016	4.0	0.3

Riprap Design in Channel Bends

Equations:

Superelevation, $\Delta d = V^2 T / (g R_c)$

Length of Protection beyond bend, $L_p / R = 0.736 (R^{1/6}) / n_b$

T = top width of channel (m)

g = acceleration of gravity (m/s^2)

R_c = curve radius (m)

R = hydraulic radius (m)

n_b = Manning's roughness in Bend

Bend No.	R_c	R	n_b	V (m/s)	T (m)	Δd (m)	Δd (in)	L_p (m)	L_p (ft)
1	69	0.49	0.035	2.1	7.5	0.0489	1.92	9.15	30
2	40	0.49	0.035	2.1	7.5	0.0843	3.32	9.15	30
3	43	0.49	0.035	2.1	7.5	0.0784	3.09	9.15	30

**Riprap shall extend ~30 feet beyond the point tangent to the curve on each curve. Superelevation will have little or no effect on channel geometry; berms along channel edge will provide protection against any spillage due to superelevation.*



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CIVIL ENGINEERING & CONSTRUCTION SERVICES

MEMORANDUM

**To: Charlotte Rodrigues, P.E.
Don McGhie**

**Department of Water and Power
City of Los Angeles
300 Mandich Street
Bishop, CA 93514**

From: Shawn Jenkins, P.E.

Date: 5/31/06

Subject: White Mountain Estates

Please accept this memorandum summarizing our meeting held in your office on March 30, 2006. Those in attendance included, Don McGhie, Senior Real Estate Officer (DWP), Charlotte Rodrigues, Engineering Services Manager (DWP), Gerry Jensen (ESE) and Shawn Jenkins (ESE). The issue discussed included the requirement of the installation of oil/water separators within the development immediately prior to the discharge point (DPW correspondence dated April 14, 2005).

As a result of our meeting it was decided that water would not be discharged onto DWP property at the Tungsten Road/ Tuolumne Road intersection. It was agreed to direct drainage across the north leg on the Tungsten Road/ Tuolumne Road intersection through a culvert continuing in the roadside ditch and crossing the north leg of the Tungsten Road/ Redwood Road intersection through a culvert and allowing the discharge to follow the natural drainage course off-site. This change would eliminate the need for oil water separators on this project. Please contact Gerry Jensen at (775) 790-8748 or Shawn Jenkins at (775) 828-7220 if this meeting summary does not reflect out discussions.

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CIVIL ENGINEERING & CONSTRUCTION SERVICES

MEMORANDUM

**To: John Langford
Bear Engineering
P.O. Box 657
Bridgeport, CA 93517**

From: Shawn Jenkins, P.E. 

Date: 6/09/06

**Subject: Drainage Plan Submittal Comments
White Mountain Estates**

This is in response to Mono County's review of the draft Tentative Tract Map dated February 17, 2006. Eastern Sierra Engineering (ESE) is responsible for the drainage portion of the improvements. Attached are revised copies of the drainage plans, drainage report & calculations addressing comments #18-#24 of Mono County's review. Revisions made to the drainage portion of the project are summarized below:

#18-Comment #18 suggests holding the drainage easement widths consistent at 30 feet. The revised plans show the easements at 30 feet except for a portion within Lot 33, which requires additional width due to the roadway crossing. The drainage channel is designed to conform to the natural topography as much as possible, thus limiting unnecessary grading activity. To obtain this, it was necessary to offer a 40-foot easement in this area, as shown on the plans.

#19-In response to comment #19, these flows are handled naturally and are not anticipated to enter the Wadkins drainage channel (Ditch A) as addressed in the actual drainage report. The natural topography intercepts these flows, therefore manmade interception is not required.

#20-We did not alter the bearing of the drainage ditch between lots 2, 12, 13, 19, 20 and 47. The drainage ditch centerline is designed to follow the property line throughout these lots. In order to straighten the ditch the property boundaries would have to be altered.

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#21-In response to comment #21, ESE met with the Los Angeles Department of Water and Power (DWP) on March 30, 2006 in regards to offsite discharge of storm water onto their property. As a result of the meeting it was decided that water would not be discharged onto DWP property at the Tungsten Road/ Tuolumne Road intersection. It was agreed to direct drainage across the north leg on the Tungsten Road/ Tuolumne Road intersection through a culvert continuing in the roadside ditch and crossing the north leg of the Tungsten Road/ Redwood Road intersection through a culvert and allowing the discharge to follow the natural drainage course off-site. This change would eliminate the need for oil water separators, as suggested by DWP. (See attached Memo)

#22-The drainage report has been revised such that the call-outs to Ditch A referring to "See section" have been eliminated.

#23-Ditch B has been eliminated from the design and the previous Ditch C is now referred to as Ditch B. The previous Ditch B was re-analyzed and found to be of little benefit to the project.

#24-Ditch C is now referred to as Ditch B as stated above. We did not address the fencing within the easement limits. Sufficient riprap lining shall be placed in all critical areas of the ditches including the entrance into Ditch A from Ditch B. We anticipate the riprap lining shall be sufficient to protect the sideslopes and prevent excessive scour.

If you have any questions regarding the revised drainage plans please contact me at (775) 828-7220.

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